Cable Supported Bridges
Cable Supported Bridges

Concept and Design, Third Edition

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Preface to the Third Edition

The decision to prepare a manuscript for a book titled CABLE SUPPORTED BRIDGES was taken by Niels J. Gimsing in 1980 following his three year affiliation as an adviser on bridge technology to Statsbroen Store Bœlt—the client organization established to design and construct a bridge across Storebælt (Great Belt) in Denmark. During the design period from 1976 to 1979, a large number of different designs for cable stayed bridges (with spans up to 850 m) and suspension bridges (with spans up to 1800 m) were thoroughly investigated and it was during that period the idea matured to write a book covering both cable stayed bridges and suspension bridges. The chance to prepare the manuscript came in 1979 when the Danish Government decided to postpone the construction of the Storebælt Bridge and to keep the design work at rest for a period of five years.

The manuscript for the First Edition was completed in 1982 and the book was published in 1983.

The decision to prepare a manuscript for a Second Edition was taken in 1994 when Niels J. Gimsing was involved in the design of both the 1624 m main span of the Storebælt East Suspension Bridge and the 490 m main span of the Øresund cable stayed bridge. Both bridges were under construction during the writing of the manuscript (from 1994–1996) and so useful information on construction issues could be collected.

The Second Edition was published in 1997; fourteen years after the First Edition appeared.

The Second Edition was sold out from the publisher after only 5 years on the market, so a Third Edition became desirable, and initially it was anticipated that this would be just a simple updating of the Second Edition. However, when digging deeper into the matter it became evident that a considerable evolution had taken place during the decennium following the publishing of the Second Edition. Very notable cable supported bridges had been constructed and a number of design issues related primarily to dynamic actions had gained in prominence.

It was, therefore, realized that the Third Edition had to be more than just a simple updating of the Second Edition. To emphasize the importance of issues pertaining to dynamic actions and health monitoring it was decided that two new chapters would be added. With his years of experience within the field, Christos T. Georgakis was entrusted with this task.


Besides revisions and additions in the text it was also decided to update the figures by preparing them in electronic versions that could be more easily edited to appear in a uniform manner throughout the publication. The financial support to cover the expenses for the figure updating came from the COWI Foundation. The figures were updated by Kristian Nikolaj Gimsing.

In the process of preparing the Third Edition, highly appreciated contributions came from Professor Yozo Fujino of the University of Tokyo, on matters relating to structural health monitoring and structural control, and from Professor Francesco Ricciardelli of the University of Reggio Calabria, on matters pertaining to bridge aerodynamics. PhD student Joan Hee Roldsgaard helped greatly with the preparation of elements of Chapters 8 and 9 and for the proof correcting of the book. Our great appreciation is also extended to all those who provided pictures, figures and copyright permissions. They are too many to mention here.

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Introduction

In the family of bridge systems the cable supported bridges are distinguished by their ability to overcome large spans. At present, cable supported bridges are enabled for spans in the range from 200 m to 2000 m (and beyond), thus covering approximately 90 per cent of the present span range.

For the vast majority of cable supported bridges, the structural system can be divided into four main components as indicated in Figure 0.1:

1. the deck (or stiffening girder);
2. the cable system supporting the deck;
3. the pylons (or towers) supporting the cable system;
4. the anchor blocks (or anchor piers) supporting the cable system vertically and horizontally, or only vertically, at the extreme ends.

The different types of cable supported bridges are distinctively characterized by the configuration of the cable system. The suspension system (Figure 0.2) comprises a parabolic main cable and vertical hanger cables connecting the deck to the main cable. The most common suspension bridge system has three spans: a large main span flanked by shorter side spans. The three-span bridge is in most cases symmetrical with side spans of equal size, but where special conditions apply, the side spans can have different lengths.

In cases where only one large span is needed, the suspension bridge may have only the main span cable supported. However, to transmit the horizontal component of the main cable pull acting at the pylon tops, the main cable will have to continue as free backstays to the anchor blocks.

A single-span suspension bridge will be a natural choice if the pylons are on land or close to the coasts/river banks so that the traffic lanes will continue on viaducts outside the pylons.

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The cable-stayed system (Figure 0.3) contains straight cables connecting the deck to the pylons. In the fan system, all stay cables radiate from the pylon top, whereas parallel stay cables are used in the harp system.

Besides the two basic cable stayed systems (the fan system and the harp system), intermediate systems are often found. In the semi-fan system, the cable anchorages at the pylon top are spread sufficiently to separate each cable anchorage and thereby simplify the detailing. With cable anchorages positioned at minimum distances at the pylon top, the behaviour of the semi-fan system will be very close to that of the pure fan system.

The stay cable anchorages at the deck will generally be spaced equidistantly so in cases where the side spans are shorter than half of the main span, the number of stay cables leading to the main span will be greater than the number of stay cables leading to the side span. In that case the anchor cable from the pylon tops to the anchor piers will often consist of several closely spaced individual cables (as shown for the semi-fan system).

In the harp system, the number of cables leading to the main span will have to be the same as in the side spans. With the anchor pier positioned at the end of the side span harp, the length of the side span will be very close to half of the main span length. That might prove inconvenient in relation to the overall stiffness of the system. It can then be advantageous to position the anchor pier inside the side span harp as indicated in Figure 0.3.

The position of the anchor pier closer to the pylon can also prove favourable in a fan system, if designed with fans of equal size in the main and side spans (Figure 0.4).
For the harp system the most efficient structural system will be achieved if a number of intermediate piers can be positioned under the side span harps (Figure 0.5). This will be the preferred solution if the side spans are on land or in shallow water.

The most common type of cable supported bridge is the three-span bridge with a large main span flanked by two smaller side spans. However, especially within cable stayed bridges, there are also examples of a symmetrical arrangement with two main spans of equal size or an asymmetrical two-span arrangement with a long main span and a somewhat shorter side span (Figure 0.6). If the two spans are of equal size, it will be necessary to stabilize the pylon top with two anchor cables whereas the asymmetrical arrangement often can be made with only an anchor cable in the shorter span.

The vast majority of cable supported bridges are built with three or two spans, but in a few cases this has not been sufficient. A straightforward solution that maintains the advantages of the three-span configuration is then to arrange two or more three-span bridges in sequence, as shown in Figure 0.7 (top). In appearance, the bridge will have every second opening between pylons without a central pier and the other openings with a central anchor pier (or anchor block).

Figure 0.4  Semi-fan system with side span pier inside the fan

Figure 0.5  Harp system with intermediate supports in the side spans

Figure 0.6  Two-span cable stayed bridges

Figure 0.7  Multi-span cable supported bridges
A true multi-span cable supported bridge will consist of a number of main spans back-to-back as shown in Figure 0.7 (bottom).

In many cases, a true multi-span cable stayed bridge (bottom) will be preferable to a series of three-span bridges (top) from the point of view of appearance and function. However, from a structural viewpoint, the true multi-span arrangement presents a number of problems.

Due to the lack of anchor cables leading from vertically fixed points at the deck level to the pylon tops, the pylon must possess a considerable flexural stiffness to be able to withstand (with acceptable horizontal displacement at the top) a loading condition with traffic load in only one of the two spans adjacent to the pylon. In such a loading condition, the cable pull from the loaded span will be larger than from the unloaded span so the pylon must be able to withstand the difference between the horizontal force from the cable system in the loaded span and in the unloaded span.

In the early cable stayed bridges built from the mid-1950s to the mid-1970s, the distance between cable anchorages at deck level was generally chosen to be quite large and as a consequence each stay cable had to carry a considerable load. It was therefore necessary to compose each stay of several prefabricated strands joined together (Figure 0.8, left).

It was necessary to let the multi-strand cable pass over the pylon on a saddle as the space available did not allow the splitting and individual anchoring of each strand, and at the deck the anchoring of the multi-strand cable made it absolutely necessary to split it into individual strands.

In modern cable stayed bridges, the number of stay cables is generally chosen to be so high that each stay can be made as a mono-strand. This will ease installation, and particularly replacement, and it will render a more continuous support to the deck (Figure 0.8, right).

With the multi-cable system it will be possible to replace the stays one by one if the deck is designed for it, which will often be required in the Design Specifications. The advantages gained in relation to erection, maintenance and replacement have to some extent been set against an increased tendency for the stays in a multi-cable system to suffer from wind-induced vibrations.

Besides the configuration of the cables, cable supported bridges can also be distinguished by the way the cable system is anchored at the end supports. In the self-anchored system, the horizontal component of the cable force in the anchor cable is transferred as compression in the deck, whereas the vertical component is taken by the anchor pier (Figure 0.9, left). In the earth anchored systems, both the vertical and the horizontal components of the cable force are transferred to the anchor block (Figure 0.9, right).

In principle, both earth anchoring and self-anchoring can be applied in suspension bridges as well as in cable stayed bridges. However, in actual practice, earth anchoring is primarily used for suspension bridges and self-anchoring for cable-stayed bridges.

For the suspension bridges, self-anchoring is especially unfavourable in relation to structural efficiency and constructability. In modern practice, self-anchored suspension bridges are therefore only seen when the decision to use the system is taken by people without structural competence and who are not concerned about construction costs.

In the transverse direction of the bridge, a number of different solutions for the arrangement of the cable systems can be found. The arrangement used traditionally in suspension bridges comprises two vertical cable planes supporting the deck.
along the edges of the bridge deck (Figure 0.10). In this arrangement (which is also seen in many cable stayed bridges), the deck is supported by the cable systems both vertically and torsionally.

In cases where the bridge deck is divided into three separate traffic areas, e.g. a central railway or tramway area flanked by roadway areas on either side, the two vertical cable planes might be positioned between the central area and the outer areas (Figure 0.11, left). This arrangement is especially attractive if the central area is subjected to heavy loads that would induce large sagging moments in the transverse girders if the cable planes were attached along the edges of the bridge deck. On the other hand, with the cable planes moved in from the edges towards the centre of the deck, the torsional support offered by the cable system will be drastically reduced. A more moderate displacement of the cable planes from the edges of the deck is found in bridges with cantilevered lanes for pedestrians and bicycles (Figure 0.11, right).

The application of more than two vertical cable planes (Figure 0.12) was seen in some of the large American suspension bridges from the end of the nineteenth century and the beginning of the twentieth century. In bridges with a wide

![Figure 0.9](image1.png)

**Figure 0.9** Connection between the side span cable and the anchor pier/block in a self-anchored system (left), and in an earth-anchored system (right)

![Figure 0.10](image2.png)

**Figure 0.10** System with two vertical cable planes attached along the edges of the bridge deck

![Figure 0.11](image3.png)

**Figure 0.11** Systems with two vertical cable planes positioned between three separate traffic lanes
bridge deck, more than two cable planes could still be considered, as the moments in the transverse girders will be significantly reduced.

Only one vertical cable plane (Figure 0.13) has been widely used in cable stayed bridges. In this arrangement, the deck is only supported vertically by the cable system, and torsional moments must therefore be transmitted by the deck. Consequently, the deck must be designed with a box-shaped cross-section.

Inclined cable planes (Figure 0.14) attached at the edges of the bridge deck and converging at the top are found in cable stayed bridges with A-shaped pylons. In this arrangement the deck is supported both vertically and torsionally by the cable system.

Two inclined cable planes converging at the top can also be supported on a single vertical pylon penetrating the deck in the central reserve or in the gap between two individual box girders.
Evolution of Cable Supported Bridges

The principle of carrying loads by suspending a rope, chain or cable across an obstacle has been known since ancient times. However, it was not until 1823 that the first permanent bridge supported by cables composed of drawn iron wires was built in Geneva by the Frenchman Marc Seguin, one of five brothers who, in the following two decades, built hundreds of suspension bridges around Europe. All of these bridges were of modest size but they marked an important step on the way to the more impressive structures that followed.

The application of thin wires in the main load-carrying elements gave rise to a number of problems especially in relation to durability, as an efficient method for corrosion protection had not been found at that time. Therefore, some of the leading engineers preferred to construct suspension bridges with the main load-carrying elements, the catenaries, composed of pin-connected eye-bars forming huge chains.

This principle was applied by the British engineer Thomas Telford in the world’s first bridge to cross a strait used by ocean-going vessels, the Menai Bridge between the British mainland and the Isle of Anglesey (Figure 1.1). Opened to traffic in 1826, this bridge had its 176 m long main span supported by chains assembled from wrought iron eye-bars, each with a length of 2.9 m.

Figure 1.1 The suspension bridge across the Menai Strait (UK)
The chain support was generally preferred by the British engineers of the nineteenth century and a number of notable bridges were built, among these the famous Clifton Suspension Bridge, initially designed by Isambard Kingdom Brunel, but not actually constructed until after his death. The bridge was opened to traffic in 1864 and it comprised a main span of 214 m – an impressive span, considering that the strength-to-density ratio of the wrought steel in the chains was less than one-fifth of the ratio of modern cable steel (Figure 1.2).

To erect the eye-bar chains, a temporary footway had to be established between the supporting points on the pylon tops and at the anchor blocks. In the case of the Clifton Suspension Bridge, this temporary footway was supported by wire ropes, so the principle of cable support was actually applied, although only in the construction phase.

A most unusual bridge based on application of eye-bar chains is the Albert Bridge across the Thames in London (Figure 1.3). The bridge was built from 1871 to 1873 and it is characterized by combining the cable stayed and the suspension system. A part of the deck load is transferred to the strong top chain through hangers and the rest is carried by a number of straight chains radiating from the pylon tops. The system is statically indeterminate to such a degree that it was impossible with the available tools to calculate forces and moments to get even close to the exact values. Nevertheless the bridge with its 122 m-long main span is still in service although there are restrictions on the traffic allowed to pass over it (Figure 1.4).

Chain support was also applied in a number of bridges on the European continent, but here it was to a larger extent in competition with cable supported suspension bridges. Thus the longest free span in Europe was for several decades found in the wire supported *Grand Pont Suspendu* across the Sarine Valley at Fribourg in Switzerland. The bridge was completed in 1834 and it had a main span of 273 m. In the Grand Pont Suspendu, each of the four main cables was composed of over 1000 wires, grouped in 20 strands, each assembled on the ground and lifted individually into position. The bridge was in service for almost a century until it was finally demolished in 1923.

On a global level, the Swiss span record was beaten in 1849 by the completion of the Wheeling Suspension Bridge across the Ohio River in the USA. This bridge had a main span of 308 m, carried by a total of 12 parallel-wire cables, six on either side of the roadway.

The Wheeling Bridge is still in existence, although not in its original version. Five years after its completion, in 1854, a violent gale blew the bridge down. Subsequently it was reconstructed and later, in 1872, further strengthened by a fan-shaped system of stays. The principle of strengthening the suspension system with stays was originally introduced during the construction of the suspension bridge across the Niagara Gorge. This bridge was designed by the famous bridge designer John A. Roebling, who was born in Germany but emigrated to the United States of America at the

![Figure 1.2 Clifton Suspension Bridge (UK)]
The Niagara Bridge was constructed in the period from 1851 to 1855 and it was the first major suspension bridge to have air-spun wire cables, a system invented by Roebling.

The span of the Niagara Bridge was not quite as long as for the largest suspension bridges of that time but, due to the fact that the bridge carried both a railroad track and a roadway, its span of 250 m was still a very impressive achievement. As a most unusual feature the truss of the Niagara Bridge had the railroad track on the upper deck and the roadway on the lower, inside the two trusses.

Another unusual feature of the Niagara Bridge was the use of wood in the truss. This might today seem to be an awkward combination of structural materials but it must be remembered that in the early days of railroad building in North America, wood was the preferred material for bridges across rivers and gorges. For the Niagara Bridge, the application of a wooden truss resulted in a relatively short lifespan as the bridge had to be replaced in 1897 after 42 years of service.
The largest of Roebling’s bridges completed during his lifetime, the Cincinnati–Covington Bridge across the Ohio River, was completed in 1866 with a record-breaking span of 322 m (Figure 1.5). In this bridge he tested many advanced features before they were adopted in his most sublime achievement: the design for the Brooklyn Bridge across the East River in New York.

**Brooklyn Bridge**

The Brooklyn Bridge across the East River between Manhattan and Long Island (Figure 1.6) is justifiably regarded as the ancestor of all modern suspension bridges and it was to a large degree detailed by Roebling before his death in 1869 shortly after the start of construction of this, the greatest bridge of his career. Opened to traffic in 1883, the Brooklyn Bridge had a centre span of almost 500 m (486 m) and side spans of 286 m, i.e. a total cable supported length of 1058 m.
Based on his experience during design and construction of several suspension bridges, and through his investigations into accidents such as the collapse of the Wheeling Bridge in 1854, Roebling had acquired a profound understanding of the aerodynamic problem. This is clearly indicated in his own description of the Brooklyn Bridge concept:

But my system of construction differs radically from that formerly practised, and I have planned the East River Bridge [as the Brooklyn Bridge was initially called] with a special view to fully meet the destructive forces of a severe gale. It is the same reason that, in my calculation of the requisite supporting strength so large a proportion has been assigned to the stays in place of cables.

This description proves that Roebling knew very well that a cable stayed system is stiffer than the suspension system, and the fact that the stays of the Brooklyn Bridge carry a considerable part of the load can be detected by the configuration of the main cable having a smaller curvature in the regions where the stays carry a part of the permanent load than in the central region, where all load is carried exclusively by the main cable.

The efficiency of the stay cables (Figure 1.7) is clearly demonstrated by the following remark by Roebling: ‘The supporting power of the stays alone will be 15,000 tons; ample to hold up the floor. If the cables were removed, the bridge would sink in the center but would not fall.’

Roebling had started his engineering career at a time when the design of bridges was still more of an art, requiring intuition and vision, than a science. Therefore, he had to acquire a profound understanding of the structural behaviour of cable supported bridges through observations and by experience. He gradually learned how to design structures of great complexity, as he could combine his intuitive understanding with relatively simple calculations, giving adequate dimensions for all structural elements.

In the case of the Brooklyn Bridge, the system adopted is one of high indeterminateness as every stay is potentially a redundant. A strict calculation based on the elastic theory with compatibility established between all elements would involve numerical work of an absolutely prohibitive magnitude, but by stipulating reasonable distributions of forces between elements and always ensuring that overall equilibrium was achieved, the required safety against failure could be attained.

After Roebling, the next generation of engineers was educated to concentrate their efforts on the calculations, which required a stricter mathematical modelling. As systems of high statical indeterminateness would involve an insuperable amount of numerical work if treated mathematically stringently, the layout of the structures had to be chosen with due respect to the calculation capacity, and this was in many respects a step backwards. Consequently, a cable system such as that used in the Brooklyn Bridge had to be replaced by much simpler systems.

The theories available for the calculation of suspension bridges in the second half of the nineteenth century were all linear elastic theories, such as the theory by Rankine from 1858, dealing with suspension bridges where the deck comprised
two- and three-hinged girders. The theory was the first to take into account rationally the interaction between the cable and the deck. Later, in 1886, the linear elastic theory was further developed by Maurice Levy in his paper, ‘Mémoires sur le calcul des ponts suspendus rigides’.

**Williamsburg Bridge**

The trend to let the calculations influence the layout of the structure is clearly seen in the Williamsburg Bridge, the second bridge to span the East River in New York (Figure 1.8). Opened to traffic in 1903, this bridge had a main span of 488 m, just 2 m more than the Brooklyn Bridge.

The structural system with unsuspended side spans and only one well-defined, simply supported suspended main span without any additional stays, clearly shows the strive towards a practicable mathematical model. Also, the extreme depth of the stiffening truss, one-fortieth of the span, can be seen as a result of the attempt to match the behaviour of the real structure to agree with the mathematical model that took only linear elastic effects into account (i.e. neglecting the change of geometry due to node displacements).

That the final result will often be less satisfactory when calculations govern the design is well demonstrated by the Williamsburg Bridge, especially in comparison with the Brooklyn Bridge. In his book, *Bridges and Their Builders* [57.1], D. B. Steinman wrote about the Williamsburg Bridge:

> With the ungainly tower design and the excessively deep trusses, the structure presents an appearance of angularity and clumsiness. It marked one extreme of the swing of the pendulum; thereafter there was a reversal of trend, toward progressively increasing slenderness and grace in the design of suspension bridges.

One feature that the Brooklyn Bridge and the Williamsburg Bridge has in common is the arrangement of the main cables at midspan in relation to the stiffening truss. In both bridges, the cables pass beneath the top chord of the truss and are led down to the bottom chord at the centre of the main span. This arrangement is very well justified from an economic point of view as the height of towers and the length of hangers, for a given sag of the main cable, are reduced by a distance equal to the depth of the truss, as illustrated in Figure 1.9. In modern suspension bridges the main cable will in general be positioned

![Figure 1.9](image)

*Figure 1.9* Position of the main cable in relation to the stiffening truss: (left) position favoured in the early suspension bridges; (right) position preferred today
entirely above the deck, which undoubtedly is preferable with regards to the appearance, as the cable curve is more easily perceived. Also, in modern bridges with slender decks, the savings would be smaller than in the Williamsburg Bridge where the truss depth corresponds to as much as 25% of the main cable sag.

### Manhattan Bridge

The third suspension bridge to span the East River was the Manhattan Bridge designed by L. S. Moisseiff and opened to traffic in 1909 (Figure 1.10).

In the evolution of cable supported bridges, the Manhattan Bridge is notable for the fact that it was the first major suspension bridge to be analyzed by the so-called ‘deflection theory’ which had been developed by Professor Melan in Vienna in 1888. The deflection theory is a nonlinear elastic theory that takes into account the displacements of the main cable under traffic load when calculating the bending moments in the stiffening truss. Thus, equilibrium is established more correctly for the deflected system than for the system with the initial dead load geometry.

To get a phenomenological understanding of the deflection theory, a suspension bridge main span subjected to traffic load in the left half of the span may be considered. As indicated in Figure 1.11, the funicular curve of the applied dead plus traffic load does not coincide with the cable curve of the dead load condition, so moments will be induced in the deck.

With a linear elastic theory based on the assumption that the change in geometry due to deflections caused by the applied traffic load can be ignored, the moments to be taken by bending in the deck can be expressed by $H_e$, where $H$ is the horizontal force (related to the funicular curve) and $e$ is the vertical distance from the cable axis to the funicular curve.

The moments induced in the stiffening truss will be positive in the span half with traffic load and negative in the remaining part of the span. This means that the deck will deflect into an S-shape, as indicated at the bottom of Figure 1.11.

However, due to the hangers linking the deck to the main cable, the deflection of the deck will cause a change in the geometry of the main cable. In Figure 1.12, the full line indicates the shape of the cable when deflections of the deck are taken into account. It will be seen that the cable moves towards the funicular curve, and as equilibrium must exist in the deflected system, the real moments in the deck will be represented by the horizontal force $H$ multiplied by the vertical distance $e - \delta$ from the funicular curve to the distorted cable.

When taking into account the nonlinear elastic effect related to the displacement of the cable, the bending moments in the deck will be reduced, often to less than half of that found by a linear elastic theory. Actually, there are no limits to the reduction that can be achieved, as a suspension bridge with a very slender deck and therefore insignificant flexural stiffness will deflect under asymmetrical loading until the displaced cable and the funicular curve coincide. Then $e - \delta = 0$ which
implies $M = 0$. This also follows from the fact that the funicular curve by definition is the curve followed by a perfectly flexible string subjected to the action of the applied load.

As equilibrium can be attained without any stiffness of the deck, the deflection theory does not assure a minimum flexural stiffness – in contrast to a linear elastic theory. However, in the early applications of the deflection theory the authorities

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**Figure 1.11**  Moments in the deck when assuming equilibrium of the system with the dead load geometry (linear elastic or ‘elastic’ theory)

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**Figure 1.12**  Moment in the deck when assuming equilibrium of the system in the deflected state (nonlinear elastic or ‘deflection’ theory)
specified a minimum depth of the stiffening truss in the interval from one-sixtieth to one-ninetieth of the span length and thereby actually introduced a lower limit for the bending stiffness.

At the time when the deflection theory was introduced, the calculation capacity was limited so the solution procedure for the nonlinear differential equation was complicated and tedious for the practising engineer. Consequently, simplifications had to be introduced in the form of charts, tables, and correction curves by which the results of the simpler, linear elastic theory could be corrected to approximate those of the deflection theory.

Besides the new analytical approach, the Manhattan Bridge also introduced several new construction techniques, such as pylon erection by vertically travelling derrick cranes, and cable wrapping by a self-propelling machine.

After the opening of the Manhattan Bridge, only modest progress was made in the design of cable supported bridges for a period of more than 20 years, although some bridges with spans slightly exceeding the span of the Manhattan Bridge were constructed.

**George Washington Bridge**

Then, in 1931, came a suspension bridge which almost doubled the free spans of all previous bridges: the George Washington Bridge across the Hudson River (Figure 1.13). With a main span of 1066 m, this was the first bridge to span more than 1 km between supports.

Designed by O.H. Ammann, the George Washington Bridge was planned from the beginning to have two decks with a roadway on the upper deck and tracks for commuter trains on the lower deck. However, due to the economic Depression at the end of the 1920s, the original project was reduced so that only the upper deck was constructed initially. Thus, from the beginning the bridge was virtually unstiffened as only roadway stringers with insignificant bending stiffness were present in the longitudinal direction at deck level.

Despite the absence of a genuine stiffening truss, the George Washington Bridge proved to be adequately stable due to the large dead load from the heavy concrete floor and to the great width of the roadway with eight traffic lanes. Also, the short side spans, with a length of less than one-sixth of the main span, increased the efficiency of the cable system and compensated thereby to a certain extent for the lack of flexural deck stiffness.

In the early suspension bridges like the Brooklyn Bridge, the Williamsburg Bridge, and the Manhattan Bridge, four vertical cable planes were arranged across the whole width of the bridge to reduce the spans of the cross beams and establish a direct transmission of the load from the roadway to the cable systems. In the George Washington Bridge, the cable planes were to be positioned in pairs outside the edges of the roadway to achieve a continuous bridge floor across the whole width. The choice of four cable planes was mainly based on considerations regarding the spinning of the

![George Washington Bridge across the Hudson River (USA)](image)
cables, as working on four cables instead of two would speed up the erection considerably. This was especially important as very large cable cross sections were required to carry the load from the wide bridge decks.

The cable system, the anchor blocks and the pylons were designed for the full double-deck structure and they were not reduced in size when the lower deck was initially omitted.

**The three-dimensional deflection theory**

An interesting development in the process of analyzing suspension bridges appeared in 1932 when L. S. Moisseiff and F. Lienhard presented a theory for the calculation of suspension bridges under lateral load [32.1]. Actually the theory can be regarded as an extension of the nonlinear elastic deflection theory (that had so far only been developed for vertical in-plane loading) to cover horizontal forces also. Thus, the inclination of the cable planes caused by the lateral deflection of the deck could now be taken into account when calculating the moments and shear forces in the horizontal wind girder.

The new theory led to a substantial reduction of the lateral load to be carried by the deck itself, and the reduction became more and more pronounced with increasing slenderness of the wind girder. It would even be possible to create lateral equilibrium without any wind girder at all.

It earlier was mentioned how the two-dimensional deflection theory developed by Melan had removed the lower bound for the flexural stiffness of the deck in the vertical direction, and now the extension of the deflection theory to cover the three-dimensional behaviour implied that a lower bound for the lateral flexural stiffness also disappeared.

In the hands of engineers deprived of the intuitive understanding found in the previous century, and now trained to trust blindly the results of the calculations, these analytical achievements could, and should, lead to serious mistakes.

The 1930s became a decade of great achievements in the field of cable supported bridges in the United States: the George Washington Bridge was followed by such impressive structures as the San Francisco–Oakland Bay Bridge, designed by Moisseiff, and the Golden Gate Bridge, designed by J. B. Strauss.

**San Francisco–Oakland Bay Bridge**

The San Francisco–Oakland Bay Bridge actually consists of two bridges, the East Bay Crossing from Oakland to the small island of Yerba Buena, and the West Bay Crossing from that island to San Francisco. However, in relation to the topic of cable supported bridges, the original East Bay Crossing is not relevant. But the West Bay Crossing consists of twin suspension bridges placed end to end with a separating anchor pier at the centre (Figure 1.14).

Each of the two suspension bridges has a main span of 704 m and side spans of 352 m, e.g. the side span length is exactly half of the main span length. This means that the central double span with the anchor pier has the same dimensions in the

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*Figure 1.14  The West Bay crossing of the San Francisco–Oakland Bay Bridge (USA)*
superstructure as the adjoining main spans. For this reason, the heavy anchor pier might at first seem unnecessary: why an anchor pier in the central span when the adjoining spans do not need any?

Considering at first only a constant load such as dead load, the anchor pier is actually inactive as the horizontal component of the cable force is constant from end anchorage to end anchorage. But in the case of traffic load applied to only one of the two main spans, the horizontal force in that part of the double bridge will be increased. The anchor pier must therefore be able to resist the difference between the horizontal force from the span with dead load plus traffic load and the horizontal force from the span with dead load only.

During the preliminary investigations, solutions without the central anchor pier were studied [33.1] but none of these more unconventional solutions reached the stage of detailed planning (Figures 3.136 and 3.166).

The Bay Bridge was from the beginning constructed with two floors, at the upper level with a six-lane roadway for cars and on the lower level a three-lane roadway for heavy traffic (trucks and buses) as well as two tracks for commuter trains. Despite the heavy loading from the two decks, the Bay Bridge could be made with only two main cables due to the smaller spans. In the early 1960s, the Bay Bridge was converted to carry only automobile traffic with five lanes at each level (Figure 1.15).

Besides the problems facing the engineers in designing the superstructure of the Bay Bridge, problems related to the substructure were also of an unknown magnitude as the deepest pier had to go down to a depth of 73 m below water level. Despite these difficulties the Bay Bridge was built in only 40 months with actual construction beginning in July 1933 and opening to traffic in November 1936. Even today this stands as a most remarkable achievement.

**Golden Gate Bridge**

Coinciding with the construction of the Bay Bridge was that of the Golden Gate Bridge where construction started in January 1933 and terminated in May 1937 (Figure 1.16). The Golden Gate Bridge has a main span of 1280 m, 20% more than the George Washington Bridge. Despite this, the bridge could be made with only two main cables, each 930 mm in diameter, compared to the four main cables each 910 mm in diameter used in the George Washington Bridge. The reason for this was partly that the Golden Gate Bridge had only a bridge floor at one level (and without any provisions for later additions) and partly that the sag ratio of the Golden Gate Bridge main cables was larger than for the George Washington Bridge.

The stiffening truss of the Golden Gate Bridge represented an extreme in slenderness as the depth-to-span ratio was only 1:168. At the same time, the space truss comprised only three plane trusses, two vertical under the cable planes and one horizontal directly below the bridge floor. This configuration resulted in an insignificant torsional stiffness of the total truss section, but at the time when the Golden Gate Bridge was designed, the importance of torsional stiffness for achieving aerodynamic stability was not fully appreciated.
Tacoma Narrows Bridge

A few years later the extreme slenderness of the Golden Gate Bridge was substantially surpassed by the next major suspension bridge on the American West Coast: the Tacoma Narrows Bridge.

This bridge, with a main span of 853 m, had the deck made up of plate girders with a depth-to-span ratio of only 1:350. This extreme slenderness was actually the ultimate result of the designer Moisseiff’s application of the deflection theory, which – as has already been described – gave ever decreasing bending moments as the flexural stiffness was reduced. Besides the small depth-to-span ratio, the width-to-span ratio of 1:72 (compared to 1:47 for the Golden Gate Bridge and 1:33 for the George Washington Bridge) also went beyond previous practice. The extreme slenderness of the wind girder was also made possible by Moisseiff’s own extension of the deflection theory to cover the three-dimensional behaviour. Furthermore, the deck of the Tacoma Bridge had virtually no torsional rigidity as only one lateral bracing was present. Despite the extremeslenderness of the deck, the bridge possessed an adequate safety margin against the action of the traffic load and the static wind pressure (drag), when taking full advantage of the nonlinear effects.

In less than 40 years from the Williamsburg Bridge of 1903 to the Tacoma Narrows Bridge of 1940, the pendulum had swung from one extreme to the other with a reduction of the relative girder depth by a factor of almost 10. However, shortly after the opening of the Tacoma Bridge, nature gave a clear demonstration of the fact that the trend towards increasing slenderness had gone too far.

Right from its opening, the bridge had shown a tendency to oscillate in the wind, but during the first four months these oscillations were vertical, with no twist of the cross section, and the oscillations were always damped down after reaching an amplitude of about 1.5 m.

Then, after a few months in service, following the breaking of inclined tie cables that had prevented mutual displacements between the deck and the main cables at midspan, the type of oscillation suddenly changed. The oscillations then took the form of twisting movements with the main span oscillating asymmetrically in two segments with a node at midspan (Figure 1.17). The torsional movements became more and more violent with a tilting of the roadway at the quarter points from $+45^\circ$ to $-45^\circ$. After approximately one hour of these violent self-excited oscillations, caused by negative damping of the aerodynamic forces, the hangers began to break in fatigue at the sockets and a large portion of the deck fell into the water.

During the final oscillations of the Tacoma Bridge the wind speed (18 m/s) was by no means extreme, and far below the maximum wind speed the bridge had been designed to withstand. However, when analyzing the structure for the action of wind, only a static pressure had been considered – and in this respect, the bridge was completely safe.